PARSONS ISLAND

COASTAL ENGINEERING INVESTIGATION

PRELIMINARY STUDY

FINAL REPORT

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EXECUTIVE SUMMARY

This Coastal Engineering Investigation report for the Maryland Port Administration is prepared to provide information on the Parsons Island site being considered for construction of a habitat restoration site using dredged material. The site would be used for placement of dredged material from the Baltimore Harbor approach channels east of the North Point/Rock Point line in the Patapsco River. This report addresses the coastal engineering aspects of the proposed project. Geotechnical engineering and dredging engineering components are being simultaneously studied and are reported separately by others.

The report addresses two major elements:

- Evaluation of existing available data pertaining to environmental site conditions and specifically related to coastal engineering aspects of design
- Design of the containment dikes as regards armor protection and structure height

ENVIRONMENTAL SITE CONDITIONS

A summary of site conditions relevant to this study is provided below:

- **Bathymetry and Topography.** Water depths are relatively shallow around the island within the proposed project area (between 0 and –5 ft MLLW). The average water depths that the dikes would be constructed in are –3 ft MLLW for Alignment No. 1 and –4 ft MLLW for Alignment No. 2. Water depths in the deeper portions of Eastern Bay near the island range from about -18 to -45 ft MLLW.

- **Wind Conditions.** Design winds for the site are developed from data collected at Baltimore-Washington International (BWI) Airport. The BWI wind data are presented as fastest mile winds which are defined as the highest recorded wind speeds that last long enough to travel one mile during a 24 hour recording period. These wind data are used to develop wind speed-return period relationships based on a Type I (Gumbel) distribution. Design wind speeds are calculated for return periods ranging from 5 to 100 years for eight wind directions to which the site is exposed, including the direction with the longest fetch (southwest).

- **Water Levels.** Normal water levels at the site are dictated by astronomical tides. Mean tide level ranges between 0.7 and 0.9 ft above MLLW. Design water levels for the project area are dominated by storm surge which for a 100-year return period can be as high as 7.5 ft above MLLW.

- **Wave Conditions.** The highest waves for the site approach from the southwest direction. Shallow bathymetry in the vicinity of the site require calculation of nearshore wave spectra. Predicated peak spectral wave period and significant offshore, significant nearshore and maximum nearshore wave heights for 5-year, 35-year and 100-year storms with winds from the southwest direction are tabulated below.

Waves for Five, Thirty-five and One Hundred Year Return Period for Maximum Fetch Direction (SW)

Return Period (years)	Peak Spectral Wave periood (sec)	Offshore Significant Wave Height (ft)	Nearshore Significant Wave Height (ft)	Nearshore Maximum Wave Height (ft)
5	5.0	6.1	3.6	6.1
35	6.0	9.6	4.3	7.5
100	6.6	12.0	5.2	9.0

- **Currents.** Currents in the project area are weak with a maximum velocity of 0.3 ft per second.

- **Site Soil Characteristics.** Results of the preliminary study conducted by E2CR indicate that the underlying soil is predominately sandy (silty sand and clayey sand) and has significant strength to support the dikes needed to contain the dredged material.

COASTAL PROTECTION DIKE DESIGN

Preliminary cross-sections are developed for coastal protection of the containment dikes. Cross-sections varied primarily in accordance with wave exposure and foundation conditions.

- **General Conditions for Dike Design**
  - Designs are based on 35-year return period storm conditions
  - Designs incorporate a 3:1 side slope
  - Dike heights are based on (1) allowable overtopping for an unarmored crest and (2) an allowance for settlement
  - Stone sizes are computed using the Van der Meer method
  - Above grade toe protection is used
  - Core is constructed using sand
  - A crushed stone roadway having a width of 20 ft is located on the structure crest.

- **Dike Section**

Dike Section 1 for Parsons Island has a crest of +8.0 ft MLLW, and includes two layers of 600 pound armor stone, two layers of 60 pound underlayer stone overlaying a geotextile that separates the stone revetment from the dike core.

Dike Section 2 for Parsons Island has a crest of +10.5 ft MLLW, and includes two layers of 3,000 pound armor stone, two layers of 250 pound underlayer stone overlaying a geotextile that separates the stone revetment from the dike core.

- **Breakwaters**

Both alternative concepts for the Parsons Island proposed project include segmented offshore breakwaters and a shore-connected breakwater on the eastern side of the island. Crest elevation would be +4 ft MLLW, with two layers of 600 pound armor stone overlaying a 60 pound core layer.

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1. INTRODUCTION

1.1 Purpose and Scope

The purpose of this Coastal Engineering Investigation report (CEI) is to prepare a preliminary coastal engineering analysis for the Parsons Island site. The analysis uses available data on bathymetry, water levels, and wind conditions to hindcast waves for the island. Wave conditions are used to prepare conceptual designs for dikes that would be used to contain dredged material for habitat creation. An evaluation of existing current and sediment data is used to support the conceptual design, and to support preliminary dredging engineering and related efforts.

1.2 Project Description

The project consists of preparing preliminary reconnaissance studies on the potential for Parsons Island to be used as a beneficial use and habitat restoration site. This preliminary assessment includes a literature search and review of environmental, geotechnical, coastal, and site and dredging engineering analysis for the possible beneficial use and habitat restoration of this island.

1.3 Project Objectives

The primary objective of this preliminary study is to provide sufficient information as to feasibility of using the island for beneficial use, and whether further evaluation is warranted.

2. ENVIRONMENTAL SITE CONDITIONS

2.1 General

Parsons Island, being studied as a potential site for beneficial use of dredged material, is located in the northern portion of the Chesapeake Bay. Parsons Island is at approximately 38° 54' N latitude and 76° 15' W longitude (Maryland State Plane Coordinates N 450,000 E 1,425,000) as shown in Figure 2-1. Parsons Island is located between Kent Island and the eastern shore of Maryland in Eastern Bay, approximately two miles from the eastern shore.

Site conditions germane to project design include bathymetry and topography, wind conditions, water levels, wave conditions, currents, and site soil characteristics. A discussion of each of these factors is presented in the following paragraphs.

2.2 Bathymetry and Topography

Hydrographic data were obtained from NOAA charts 12263, 12270 and 12273. Vertical and horizontal data are referenced to mean lower low water (MLLW) based on the 1960 to 1978 tidal epoch, and the Maryland State Plane, North American Datum 1983. The bathymetry in the area of Parsons Island is presented in Figure 2-2. Water depths are relatively shallow around the island within the proposed project area (between 0 and –5 ft MLLW). The average water depth that the dikes would be constructed are –3 ft MLLW for Alignment No. 1 and –4 ft MLLW for Alignment No. 2. Water depths in the deeper portions of Eastern Bay near the island range from about -18 to -45 ft MLLW.

2.3 Wind Conditions

Annual extreme windspeed data from the National Oceanic and Atmospheric Administration, National Climatic Data Center for Baltimore-Washington International (BWI) Airport, for the period 1951 through 1982, were used in estimating wind conditions for this study (NOS 1982 and NCDC 1994). The BWI data are presented in Table 2-1 as fastest mile winds which are defined as the highest recorded wind speeds that last long enough to travel one mile during a 24 hour recording period. For example, a fastest mile wind speed of 60 miles per hour would have a duration of 60 seconds, a fastest mile wind speed of 50 miles per hour would have a duration of 72 seconds, etc. The wind data presented in Table 2-1 were used to develop windspeed-return period relationships based on a Type I (Gumbel) distribution are for eight directions; namely: North (N), Northeast (NE), East (E), Southeast (SE), South (S), Southwest (SW), West (W) and Northwest (NW). Return period is defined as the average time between wind events which equal or exceed a given value. The specific return periods examined were 5, 10, 15, 20, 25, 30, 35, 40, 50 and 100 years.

Table 2-1

Annual Extreme Wind Speed Per Direction
for Baltimore-Washington International (BWI) Airport, 1951-1982
Fastest Mile Wind Speed (mph)

Year	North	Northeast	East	Southeast	South	Southwest	West	Northwest	All Directions
1951	24	41	27	34	39	29	42	46	46
1952	66	25	47	66	41	66	46	43	66
1953	20	28	22	27	34	39	47	43	47
1954	31	27	22	60	28	39	57	44	60
1955	21	43	29	28	43	53	40	43	53
1956	29	34	25	24	28	34	56	40	56
1957	29	53	35	33	33	30	46	46	53
1958	30	52	25	33	37	43	40	43	52
1959	28	26	20	27	23	38	46	43	46
1960	26	38	28	27	25	35	40	53	53
1961	45	28	28	29	24	70	41	54	70
1962	56	41	28	17	25	36	42	61	61
1963	38	32	18	34	25	28	44	60	60
1964	34	31	23	24	47	23	48	61	61
1965	36	26	28	34	36	54	44	44	54
1966	32	25	29	24	47	43	50	48	50
1967	30	29	25	39	27	46	53	43	53
1968	45	30	36	26	19	45	48	50	50
1969	28	21	20	34	26	45	45	53	53
1970	28	28	18	21	39	34	48	60	60
1971	31	45	26	18	21	41	39	58	58
1972	28	25	35	26	20	41	41	41	41
1973	40	26	26	38	26	35	49	33	49
1974	32	23	46	29	33	33	45	41	46
1975	40	26	21	24	25	38	54	45	54
1976	31	18	20	28	32	28	45	54	54
1977	32	31	19	28	26	25	49	48	49
1978	39	28	36	28	19	52	33	45	52
1979	32	25	27	36	32	32	45	47	47
1980	33	27	18	32	20	32	45	50	50
1981	24	24	19	26	23	28	41	42	42
1982	31	20	23	23	29	34	40	48	48

Note: Data adjusted to 10 meter height.

A review of the wind speed data indicate that during the 32-year period from 1951 through 1982, six wind events exceeded 60 miles per hour (mph). In order to quantify the frequency of various wind events, statistical analyses of the wind data were performed. These analyses consisted of fitting external statistical distributions through the annual extreme windspeeds for each of the wind directions and all of the directions. The wind statistics for each direction (design wind speeds) are presented in Table 2-2 in terms of fastest mile windspeeds for various return periods. Table 2-2 shows that the design windspeeds for a 25-year return period storm range from 47 mph for the east direction to 70 mph for the southwest direction. The design windspeeds presented in Table 2-2 have been used to estimate design wave conditions for the proposed concept areas in Section 2.5 of this report.

Table 2-2

Design Wind speeds per Direction and Return Period (mph)

Return Period	Direction: N	NE	E	SE	S	SW	W	NW
5	40	37	32	37	36	47	50	54
10	48	44	38	45	43	56	54	59
15	52	48	41	50	47	61	56	62
20	56	52	45	55	51	67	59	65
25	59	55	47	58	54	70	60	67
30	62	57	49	61	56	73	61	68
35	64	60	51	63	58	76	62	70
40	66	62	53	65	60	78	63	71
50	69	66	55	69	63	82	64	73
100	81	76	65	82	74	97	69	81

2.4 Water Levels

Normal water level variations in the upper Chesapeake Bay are generally dominated by astronomical tides, although wind effects and freshwater discharge can be important. Extreme water levels, on the other hand, are dictated by storm tides.

2.4.1 Astronomical Tides

Astronomical tides in the upper Chesapeake Bay are semi-diurnal. The mean tide level is between 0.7 and 0.9 ft above MLLW; the mean tidal range is between 1.0 and 1.2 ft and the spring tidal range is between 1.5 and 1.8 ft (NOS 1997). Tidal datum characteristics for locations near the project sites reported from National Ocean Service are presented in Table 2-3. The difference in elevation between MLLW and national geodetic vertical datum (NGVD) is approximately 0.3 ft for the upper bay region. MLLW will serve as the datum for this project.

Table 2-3

Astronomical Tidal Datum Characteristics for Selected Chesapeake Bay Locations (ft, MLLW)

Tidal Datum	Kent Island Narrows	Love Point	Matapeake
Mean Higher High Water (MHHW)	1.8	1.7	1.5
Mean High Water (MHW)	1.5	1.4	1.2
Mean Tide Level (MTL)	0.9	0.8	0.7
National Geodetic Vertical Datum (NGVD)	0.3	0.3	0.3
Mean Low Water (MLW)	0.3	0.3	0.2
Mean Lower Low Water (MLLW)	0.0	0.0	0.0

2.4.2 Storm Surge

Design water levels for the study site areas are dominated by storm effects (i.e. storm surge and wave setup) in combination with astronomical tide. Storm surge is a temporary rise in water level generated either by large-scale extra-tropical storms known as northeasters, or by hurricanes. The rise in water level results from wind action, the low pressure of the storm disturbance and the Coriolis force. Wave setup is a term used to describe the rise in water level due to wave breaking. Specifically, change in momentum which attends the breaking of waves propagating towards shore results in a surf zone force that raises water levels at the shoreline. A comprehensive evaluation of storm-induced water levels for several Chesapeake Bay locations has been conducted by the Virginia Institute of Marine Science (1978) as part of the Federal Flood Insurance Program. Results of this study are summarized in the water-level vs. frequency curves presented in Figures 2-3 which provide water levels in feet above NGVD for various return periods. Data in Figure 2-3 are for the closest station location for the project site, which is Matapeake. Matapeake is located at 38° 58' north latitude and 76° 21' west longitude, about 6.5 miles northwest of Parsons Island. Figure 2-3 indicates that the storm tide elevation for a 25-year return period at Matapeake is 5.2 ft MLLW and the 100-year water level is 7.5 ft MLLW. As a means of comparison, the 25-year return period elevations for Baltimore and Annapolis are 5.4 MLLW and 5.1 ft MLLW, respectively.

2.5 Wave Conditions

Parsons Island is exposed to wind-generated waves approaching from all directions. In accordance with procedures recommended by the U.S. Army Corps of Engineers (USACE), Shore Protection Manual (SPM) (USACE 1984), a radially averaged fetch distance was computed for each direction. The radial averaged fetch distances for the N, NE, E, S, SE, SW, W and NW directions for the Parsons Island site are shown in Figure 2-4 and Table 2-4. Wave conditions were hindcast along each fetch direction for the design winds presented in Table 2-3 (adjusted appropriately for duration) the water levels presented in Figure 2-3, and the mean water depths along the fetch directions as shown in Table 2-4. Specifically, waves were hindcast for eight directional design windspeeds (i.e. the design windspeeds computed for each individual direction) using methods published in the Shore Protection Manual (1984). Wave hindcast results are presented in Figure 2-5 (significant wave height, H_s) and Figure 2-6 (Peak Wave Period, T_p). These figures present a summary of H_s and T_p that provide an immediate understanding of the directions from which the highest waves and longest periods approach the site.

Table 2-4
Radial Fetch Distance and Mean Water Depth for Radially-Averaged Fetch Used for Wave Hindcasting
Parsons Island Site

Direction	Mean Distance (Miles)	Mean Water Depth (ft, MLLW)
North	3.0	14.0
Northeast	2.3	25.0
East	2.0	21.0
Southeast	3.5	12.0
South	4.7	30.0
Southwest	21.7	18.0
West	2.5	17.0
Northwest	2.5	17.0

A sea state is normally composed of a spectrum of waves with varying heights and periods which may range from relatively long waves to short ripples. In order to summarize the spectral characteristics of a sea state it is customary to represent that wave spectrum in terms of a distribution of wave energy over a range of wave periods. Having made this distribution, known as a wave spectrum, it is convenient to represent that wave spectrum by a single representative wave height and period. The wave conditions reported in Figures 2-5 and 2-6 are the significant wave height, H_s, and the peak spectral wave period, T_p,, respectively. The significant wave height, H_s, is defined as the average of the highest one-third of the waves in the spectrum. Depending on the duration of the storm condition represented by the wave spectrum, maximum wave heights may be as high as 1.8 to 2 times the significant wave height. The peak spectral period, T_p, is the wave period, which corresponds to the maximum wave energy level in the wave spectrum.

For Parsons Island, the highest waves are estimated to approach from the southwest direction. The 100-year return period wave for this direction has a significant height (H_s) of 12.0 ft and peak spectral wave period (T_p) 6.6 seconds. The 35-year return period significant wave height (H_s) is 9.6 ft and the peak spectral wave period (T_p) is 6.0 seconds.

The above wave heights represent deep water conditions some distance offshore of the proposed dike alignments. The dikes will be located in water having depths shallow enough to allow for some breaking of the waves, especially for higher return period events.

Discussions presented above indicate that waves in the deep water wave spectrum may be as much as twice the offshore significant wave height and the dike structures would be exposed to some breaking waves. The random wave analyses of Goda (1985) has been used to examine the maximum breaking and maximum significant waves which can reach the dikes. The first step in examining wave conditions for a given bottom elevation and water level is to compute the total water depth from which the maximum breaking wave height can be determined. This breaker depth, h_b, is the sum of the selected water elevation above MLLW and the bottom elevation below MLLW. The maximum breaker height which can be supported in the resulting water depth is computed using the following formulae published in the SPM (USACE 1984):

$$h_b = \frac{H_b}{B_w - A_w \dfrac{H_b}{gT^2}}$$

$$B_w = \frac{1.56}{(1 + e^{(-19.5m)})}$$

$$A_w = 43.75\,(1 - e^{-19m})$$

Where: H_b = breaking wave height at the outer edge of the surf zone
m = tangent of beach slope
h_b = breaker depth
g = acceleration due to gravity
T = spectral wave period
A_w, B_w = Breaking wave height parameters (w referring to wave)

Solution to the above equation will provide an estimate of the maximum breaker height to which the structure is subjected for a given total water depth. Goda's analyses requires the estimate of an equivalent offshore significant wave height (also referred to as the equivalent unrefracted wave height) which is computed from the maximum breaking wave height and the linear shoaling coefficient in accordance with the following equations:

$$H_s \approx \frac{H_b}{1.8}$$

$$H'o = \frac{H_s}{K_s}$$

$$K_s = \sqrt{\frac{1}{\tanh\left(2\pi \dfrac{h_b}{L}\right)\left[1 + \dfrac{\dfrac{4\pi\, h_b}{L}}{\sinh\left(4\pi \dfrac{h_b}{L}\right)}\right]}}$$

$$L = \frac{gT^2}{2\pi} \tan h\ (2\pi \frac{h_b}{L})$$

Where: H_s = approximate significant wave height at breaking

H_o' = equivalent unrefracted deepwater significant wave height

K_s = shoaling coefficient

L = local wave length

The H_{max} values are computed using the following equations published by Goda (1985):

$$H_{max} = 0.8\, k_s H_o \quad for\ \frac{h}{L_o} \geq 0.2$$

and

$$H_{max} = \text{MIN}\ [(\beta_0^* H_{o'} + \beta_1^* h), \beta_{max}^* H_{o'}, 1.8\, K_s H_{o'}] \quad for\ \frac{h}{L_o} < 0.2$$

Where:

h = water depth

L_o = deepwater wave length

$$\beta_0^* = 0.052 \left(\frac{H_{o'}}{L_o}\right)^{-0.38} e_{20.0m^{1.5}}$$

$$\beta_{max}^* = \text{MAX}\,(1.65, 0.53 \left(\frac{H_{o'}}{L_o}\right)^{-0.29} e^{2.4m})$$

$$\beta^{*1} = 0.63\, e^{3.8m}$$

Similar equations are available for computing H_S, and the results are used to compute the nearshore significant and maximum wave heights. Figure 2-7 presents the polar plot for nearshore significant wave heights at Parsons Island. Figure 2-8 presents polar the plot for nearshore maximum waves heights at Parsons Island. Figures 2-7 indicates that for Parsons Island, nearshore significant wave heights from the southwest are 3.6 and 5.2 ft for a 5-year storm and 100-year storm, respectively. Figure 2-8 indicates that for Parsons Island, the maximum depth limited (breaking) waves from the southwest are 6.1 and 9.0 ft, for the 5-year and 100-year storms, respectively.

2.6 Currents

Peak tidal current velocities in Bay east of Parsons Island are about 0.3 ft/sec (NOS 1996). These currents are considered relatively weak.

2.7 Soil Characteristics

An evaluation of the soil characteristics at the project site was performed by Engineering Consultation Construction Remediation, Inc. (E2CR 2001). The evaluation included performing soil borings, preparing soil boring profiles, identifying soil strata thickness, location and characteristics, and conducting a preliminary slope stability analysis. Results of the preliminary study indicate that the underlying soil (to –30 ft MLLW) is predominately sandy (silty sand and clayey sand) and has significant strength to support the dikes needed to hold the dredged material in place as substrate for habitat development.

3. DIKE CONSTRUCTION

3.1 Introduction

The principal components of a coastal protection dike include:

- Toe Protection
- Protective Revetment
- Berm (if included)
- Upper Slope
- Crest Area and Roadway
- Dike Core

Toe protection is normally an integral part of the revetment structure and is designed to prevent that structural component from undermining as a result of wave and/or current-induced scour. The protective revetment serves to hold the dike core in place and is often comprised of several layers of rock armoring. A berm may or may not be included in the dike cross section. Where included, a berm can be used to limit wave runup and overtopping. The berm can also be used to minimize the armoring requirements for the revetment and upper slope of the dike. Roadways are often included on dikes in order to provide access to hinterland areas and access for repairs to the dikes.

The dike geometry used for this preliminary study is comprised of toe protection, a rubble mound revetment (i.e. the side slope), a horizontal crest with a crushed stone roadway and a core constructed of sand. One of the more important variables of the dike design is the side slope which, together with the crest height, is generally dictated by soil conditions and dike construction methodologies. Based on the analyses performed for prior projects, and the geotechnical analysis performed for this project, the dike design has been determined to have an outer slope of 3 horizontal to 1 vertical (3:1) and an inner side slope of 5 horizontal to 1 vertical (5:1).

3.2 Dike Design Life

The design life selected for the containment dikes is an important factor in the overall planning. It should be noted that the project life for dike design is different than the life capacity of the site for storing dredged material. The former pertains to the life expectancy and costs of the containment dikes and is treated in this section of the report whereas the latter pertains to the period of time it takes to fill the dredged material placement site.

Previously, USACE would stipulate a project life of 50 years (ER-1110-2-1407 "Hydraulic Design of Coastal Shore Protection Projects"). This has now been superseded by the revised ER-1110-2-1407 (November 1990) which dictates that a fuller range of alternatives be studied to account for differences in cost of repair, periodic replacements and rehabilitation. The 50-year project life is consistent with the nature of routine coastal and hydraulic engineering projects, which are designed to protect large areas of rural and urban infrastructure against flooding and/or wave-induced damages. Furthermore, such projects are normally justified on the basis of a rigorous and codified economic analysis, which assures that the project benefits exceed project costs. The most rational means for selecting the project design life for Parsons Island, however, is on the basis of economics (i.e. project costs and cost effectiveness). This approach was used for the design of the Poplar Island Restoration Project dikes (GBA–M&N JV 1995), and has been used in selecting the design return periods for this project.

3.3 Dike Design Values

The dikes must be designed for a given level of hydrodynamic design conditions including winds, waves, water levels, and currents. Design conditions can be stipulated in terms of levels of risk and/or in terms of statistical return period. These two factors are related to one another and the project life through the following formula:

$$R = 1 - 1\left(1 - \frac{1}{RP}\right)^{L}$$

Where: R = risk or probability that a given condition will be equaled or exceeded
L = project life in years
RP = return period in years

The previous USACE criteria stipulate that a project should be designed for an event that has a 50% risk during a 50 year project life. Manipulation of the above formula will show that these criteria correspond to a return period of 73 years. Stated simply, the return period is the average time intervals between events of a similar magnitude. For example, a 73-year design wave would be a wave that occurs an average of once every 73 years. For this study, an optimization analysis similar to that used for Poplar Island was performed. The results indicate that a 35-year return period is optimal for design of the dikes at Parsons Island, which is comparable to the results obtained for the Poplar Island project.

3.4 Geotechnical Factors

The main geotechnical factors that should be evaluated in the design of the containment dikes are (Pilarczk 1990):

- Macro-instability of slopes due to failure along circular or straight sliding surfaces
- Settlements and horizontal deformations due to the self weight of the structure
- Micro-instability of slopes caused by groundwater seepage out of the slope face
- Piping or internal erosion due to seepage flow underneath the structure
- Liquefaction caused by erosion (flow down the side slopes) or by cyclic loading wave actions or earthquakes)
- Erosion of revetments at the outer slopes (or underwater slopes) due to instable filters or local failure of top layer elements

The phenomena most germane to the overall planning of the dike designs are: (1) slope stability which dictates maximum allowable combinations of side slopes and structure heights and (2) settlement which influences the initial and final crest elevation of the dike. The geotechnical assessment indicates that an outer structure slope of three horizontal to one vertical (3:1) is feasible (E2CR 2001). Wave runup, overtopping, armor stone sizing and toe scour protection are evaluated for a 3:1 side slope. It is noted that this side slope is the same as that used for the majority of the dike at Hart Miller Island Dredged Containment Facility (HMI DMCF) and the design for Poplar Island.

3.5 Dike Height - Wave Runup and Overtopping

One of the primary functions of the containment dikes is to protect the interior of the diked placement area against the adverse effects of high water and waves. If a high level of protection is required, the structure should have a height well above the maximum level of wave runup during storm surges. Typically, this requires setting high crest elevations for the structure. However, if some overtopping is allowed based on the nature of the site (i.e. wetlands), the design requirement can be evaluated in terms of allowable overtopping. The design then is based on maintaining the structural integrity of the dikes themselves with minimal concern for protecting the interior.

The level of protection against high water and wave attack has been defined as the return period of the storm event that balances initial dike construction capital costs with long-term operations and maintenance costs needed to repair the dike as a result of destruction from wave runup and/or overtopping waves. Wave runup, and more importantly, overtopping computations allow an objective means for evaluating the level of protection (i.e. allowable overtopping) offered by various dike height and armor protection combinations. In addition, wave overtopping computations provide a rational means for evaluating the relative risk of dike breaching and subsequent failure.

Wave runup is commonly evaluated on the basis of the composite-slope runup method outlined in the Shore Protection Manual (SPM) (USACE 1984). This approach has been critically reviewed by FEMA (1988) who found that the composite slope method provides a valid method for estimating the *mean* runup value in random waves but was lacking in its ability to predict *extreme* values of wave runup. The mean runup values computed using the FEMA composite-slope runup model are generally on the order of 2 to 4 ft above the still water level under extreme conditions (e.g. 50 to 100 year storms). Low or insignificant wave overtopping discharge values are normally computed on the basis of the mean wave runup values.

Dutch engineers have long appreciated the need to consider wave runup levels higher than the mean values in design applications and have generally used the 2% exceedence runup value to select the heights of dunes and coastal dikes. Van der Meer (1992) published the following formulae for computing the 2% runup for seawalls and dikes:

$$R_{2\%} = 1.5\,\gamma_f\,\gamma_h\,\xi_p$$

$$Maximum = 3.0\,\gamma_f\,\gamma_h$$

$$\xi_p = \tan\frac{\alpha}{\sqrt{S_p}}$$

$$S_p = \frac{H_s}{\frac{g}{2\pi} T_p^2}$$

Where: $R_{2\%}$ = 2% wave runup (wave runup exceeded only 2% of the time during a storm)

γ_f = influence factor for roughness

γ_h = influence factor for shallow water

ξ_p = breaker parameter (surf similarity parameter) based on equivalent slope

α = angle of beach and or structure slope

S_p = wave steepness

H_s = significant wave height, average of highest one-third

g = acceleration of gravity

T_p = peak spectral wave period

Van der Meer's formulae are based on an extensive series of physical model tests including several full scale tests for 3:1 slopes.

The influence of roughness based on Van der Meer (1992) is summarized as follows:

Surface Covering	**Influence Factor** (γ_f)
One Layer of Rock	0.55-0.60
Two or More Rock Layers	0.50 - 0.55

A value of 0.55 was used for the present work.

When a dike is located in shallow water, the higher waves will break before they reach the structure. In that case, the distribution of wave heights at the toe of the structure must take wave breaking into account. The influence factor, γ_f, for shallow water can be determined by the following formula:

$$\gamma_h = \frac{H_{2\%}}{1.4 H_s}$$

For a gentle foreshore slope of 1:100 the following formula can be used:

$$\gamma_h = 1 - 0.03\left[4 - \frac{h}{H_s}\right] \qquad \text{for } 1 \le h/H_s \le 4$$

Finally, the mean runup can be estimated from the 2% runup using the following formula which assumes a Rayleigh distribution:

$$\gamma_h = 1 \qquad \text{for } h/H_S \ge 4$$

While wave runup is an important overall indicator of the protection offered by coastal dikes, wave overtopping is judged to be a more objective and rationale method for estimating level of wave protection for the present work. Van der Meer (1992) presents the following formula for estimating the mean wave overtopping on coastal structures subject to random waves:

$$\frac{q}{\sqrt{gH_S^3}} = 8 \times 10^{-5} \exp\left[3.1 \frac{R_{u2\%} - R_c}{H_S}\right]$$

Where: q = mean wave overtopping discharge per unit width

R_c = dike crest freeboard (height of structure above still water)

$R_{u2\%}$ = 2% wave runup

The reliability of the above equation can be given by assuming that $log(q/\sqrt{g}H^3_S)$ has a normal distribution with a variation coefficient (the ratio of the standard deviation to the mean value) of

0.11. Reliability bands can then be calculated for various practical values of mean overtopping discharges. The 90% confidence bands have been used for the purposes of this report.

The above overtopping formula provides a means for computing wave overtopping on dikes of various geometries (i.e. structure slopes, slope breaks and crest elevations). In order to evaluate the level of protection offered by a given dike configuration, it is necessary to establish limiting values of allowable overtopping. Critical or allowable overtopping discharges have been published by the United Kingdom (UK) Construction Industry Research and Information Association (CIRIA) and the Netherlands Centre for Civil Engineering Research and Codes (CUR) (CIRIA/CUR 1991). Similar values have also been published by Goda (1985). The Goda allowable overtopping values have been used in this study and are summarized below:

Structure Type	Surface Armoring	Overtopping Rate (Liters/m·s)
Type I: Coastal Dike	Concrete on front slope, soil on crown and back slope	5
Type II: Coastal Dike	Concrete on front slope and crown, soil on back slope	20
Type III: Coastal Dike	Concrete on front slope, crown and back	50
Type IV: Revetment	No pavement on ground	50
Type V: Revetment	Pavement on ground	200

Overtopping computations were used to develop required crest elevations for construction of a dike with no armor stone on the crest or back slope. The results are summarized in the polar plots presented in Figure 3-1 for a dike having a 3:1 side slope.

According to these figures required crest elevations for Parsons Island are from the southwest direction, and range from about 8 ft MLLW for a 5-year storm to about 14 ft for a 100-year event. The lowest required crest elevations for Parsons Island are for dikes exposed to waves from the east, and range from about 4 ft to 9 ft.

3.6 Armor Stone

There are a number of methodologies available for determining armor stone requirements for dike revetments subject to wave attack. A commonly used method is based on the Hudson equation published in the SPM (USACE 1984):

$$W = \frac{\gamma_r H^3}{K_D (S_r - 1)^3 \cot(\theta)}$$

Where: W = weight of armor stone

γ_r = unit weight of the armor rock (taken as 165 pounds per cubic foot)

H = wave height to which the structure is exposed

K_D = stability coefficient

$S_r = \gamma_r / \gamma_w$

γ_w = unit weight of water (taken as 64 pounds per cubic foot)

θ= angle of structure slope

The dikes at Parsons Island will be located in relatively shallow water (primarily –3 to -4 ft MLLW), and will be exposed to a wave spectrum characterized by both breaking and non-breaking waves. The wave height used in the above equation depends on whether one is evaluating breaking or non-breaking waves. According to the SPM (USACE 1984), an H_{10} wave height, which is equal to 1.27 times the significant wave height (H_s), is used for the non-breaking wave height while the maximum depth limited wave height is used for breaking waves.

Previous studies have shown that use of the Hudson results in relatively large armor stone sizes, and a more appropriate method is to use procedures published by Van der Meer (1988). Van der Meer's equations for sizing armor stone subject to shallow water random waves are as follows:

For Plunging Waves:

$$\frac{H_{2\%}}{\Delta} = 8.7\ P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}$$

For Surging Waves:

$$\frac{H_{2\%}}{\Delta} D_{n50} = 1.4\, P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^P$$

The waves are of the surging types when:

$$\xi_{mc} \geq 6.2\, P^{0.31} \sqrt{\tan \theta}^{\,1/(P+0.5)}$$

Where: $H_{2\%}$= two percent exceedence wave height

$\Delta = S_r$

D_{n50}= mean nominal diameter of the stone = $(W/ S_r)^{1/3}$

S = structural damage level taken as 2 for 0-5% damage

N = number of waves in the storm (a value of 7000 was used)

P = structure permeability (taken as 0.1 which is typical of a revetment structure with an armor layer, under or filter layer and an impermeable core)

ξ_m= surf similarity parameter

The surf similarity parameter is defined as:

$$\xi_m = \frac{\tan(\theta)}{\sqrt{\dfrac{2\pi H_s}{g T_p^2}}}$$

Where: H_s = significant wave height

T_p= peak spectral wave period

Computations were made using Van der Meer's equations for each exposure direction. The methodology presented by Van der Meer is judged to be most applicable because it is based on random wave conditions which may include breaking and non-breaking waves. The guidance presented in the SPM (USACE 1984) are based on monochromatic (i.e. single sine wave) wave conditions. Furthermore, the SPM methodology is difficult to apply in situations where there are only a few breaking waves in the design wave spectrum. Accordingly, the Van der Meer methodology will be used as the basis for preliminary dike design. It has been incorporated into the USACE's Automated Coastal Engineering System (ACES) and has been recommended in lieu of the Hudson Equation in the USACE's EM- 1100-2-1614 "Design of Coastal Revetments, Seawalls and Bulkheads" (USACE 1985).

Figures 3-2 presents a polar plot of stone sizes (computed using Van der Meer's method for breaking and non-breaking waves) for a 3:1 side slope. For Parsons Island, required stone sizes for dike sections exposed to the southwest range from 0.66 tons for a 5-year return period to 2.4 tons for a 100-year return period. At Parsons Island the dike sections facing the eastern exposure directions require armor stone ranging from 57 pounds for a 5-year return period to 460 pounds for a 100-year return period.

The above armor stone requirements assume that the armor layer for the dike revetments will consist of two layers of placed rock. This is the normal design practice prescribed in the SPM and in many other coastal engineering references.

3.7 Scour Protection

Toe scour protection is the supplemental armoring of the bottom surface fronting a structure that prevents wave energy from scouring and undercutting it. Factors that affect the severity of toe scour include wave breaking, wave runup and rundown, wave reflection and grain size distribution of the beach or bottom materials. Toe stability is essential because failure of the toe will generally lead to failure throughout the entire structure. Toe scour is a complex process and specific design guidance has not been developed. Some general guidelines, however, have been suggested.

A berm toe apron has been selected for the project for several reasons: (1) the berm will provide greater protection to the structure from overtopping as a significant number of waves will break prior to reaching the side slope, (2) construction costs for a berm toe are generally lower than for a buried toe, (3) higher quantities of sediment can be suspended during excavation and construction of a buried toe, and (4) the construction methodology and environmental concerns associated with this project are better served by using a berm toe.

3.8 Underlayers and Filters

Revetments are normally constructed with an armor layer and one or more underlayers. Revetments often have two layers of armor and a thin underlayer overlying a geotextile built upon a core of sand or clay. Small particles beneath the geotextile should not be washed through the fabric and the underlayer stones should not be washed through the armor.

The SPM (USACE 1984) recommends that underlayer stone range of 1/10 to 1/15 of the armor weight. This results in a relatively large underlayer that has two advantages. First, a large underlayer permits surface interlocking with the armor. Second, a large underlayer gives a more permeable structure and therefore has an influence on the stability of the armor layer. For the dike design, the SPM criteria are recommended.

3.9 Dike Cross Sections

Figures 3-3 and 3-4 present the conceptual alternative dike alignments (No. 1 and No. 2, respectively) for the Parsons Island proposed project. Two different dike cross sections have been developed for the alignments based on exposure direction. Figures 3-5 and 3-6 present typical dike cross sections for along Parsons Island. The primary characteristics of the dike design are:

- Designs are based on 35-year return period storm conditions
- Designs incorporate a 3:1 side slope
- Dike heights are based on (1) allowable overtopping for an unarmored crest and (2) an allowance for settlement
- Stone sizes are computed using the Van der Meer method
- Above grade toe protection is used
- Core is constructed using sand
- A crushed stone roadway having a width of 20 ft is located on the structure crest.

Figure 3-5 shows Dike Section 1 for Parsons Island with a crest of +8.0 ft MLLW, and includes two layers of 600 pound armor stone, two layers of 60 pound underlayer stone overlaying a geotextile that separates the stone revetment from the dike core.

Figure 3-6 shows Dike Section 2 for Parsons Island with a crest of +10.5 ft MLLW, and includes two layers of 3,000 pound armor stone, two layers of 250 pound underlayer stone overlaying a geotextile that separates the stone revetment from the dike core.

3.10 Breakwaters

Both alternative concepts for the Parsons Island proposed project include segmented offshore breakwaters and a shore-connected breakwater on the eastern side of the island (refer to Figures 3-3 and 3-4). Figure 3-7 shows the typical cross-section for both types of breakwaters, which is based on a 35-year return period. Crest elevation would be +4 ft MLLW, with two layers of 600 pound armor stone overlaying a 60 pound core layer.

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 Site Conditions

This Coastal Engineering Investigation report is prepared to provide information on the Parsons Island site being considered for beneficial use of dredged material. The report addresses evaluation of existing available data pertaining to environmental site conditions and coastal engineering aspects for the design of the diked enclosure.

Water depths in the area where the dikes will be located range from 0 to –5 ft MLLW. The average water depths that the dikes would be constructed are –3 ft MLLW for Alignment No. 1 and –4 ft MLLW for Alignment No. 2.

Design winds for the site are developed from data collected at Baltimore-Washington International (BWI) Airport. Design wind speeds are calculated for return periods ranging from 5 to 100 years for eight wind directions including the direction with the longest fetch (southwest).

Normal water levels at the site are dictated by astronomical tides. Mean tide level ranges between 0.7 and 0.9 ft above MLLW. Design water levels for the project area are dominated by storm surge which for a 100-year return period can be as high as 7.5 ft above MLLW.

The highest waves for the site approach from the southwest direction. Parsons Island is relatively sheltered from wind generated waves approaching from other directions. Predicted peak spectral wave period and significant offshore, significant nearshore and maximum nearshore wave heights for the southwest direction for the 5-year storm are 5.0 seconds, 6.1 ft, 3.6 ft and 6.1 ft, respectively. Predicted peak spectral wave period and significant offshore, significant nearshore and maximum nearshore wave heights for the southwest direction for the 35-year storm are 6.0 seconds, 9.6 ft, 4.3 ft and 7.5 ft, respectively. Predicted peak spectral wave period and significant offshore, significant nearshore and maximum nearshore wave heights for the southwest direction for the 100-year storm are 6.6 seconds, 12.0 ft, 5.2 ft and 9.0 ft, respectively.

Currents in the project area are weak with a maximum velocity of 0.3 ft per second.

Results of the preliminary study by E2CR indicate that the underlying soil is predominately sandy (silty sand and clayey sand) and has significant strength to support the dikes needed to contain the dredged material.

4.2 Coastal Protection Dike Design

Preliminary cross-sections are developed for the containment dikes of the habitat restoration area. Cross-sections varied primarily in accordance with wave exposure and foundation conditions. The dike designs are based upon a 35-year return period. Dike heights are based on allowable overtopping for an unarmored crest and an allowance for settlement. Stone sizes are computed using the Van der Meer method. The designs incorporate 3:1 side slope, above grade toe protection, a core constructed of sand, and a crushed stone roadway on the structure crest.

Two dike sections have been developed for Parsons Island (refer to Figures 3-3 and 3-4). Dike Section No. 1 for Parsons Island has a crest of +8.0 ft MLLW, and includes two layers of 600 pound armor stone, two layers of 60 pound underlayer stone overlaying a geotextile that separates the stone revetment from the dike core. Dike Section No. 2 has a crest of +10.5 ft MLLW, and includes two layers of 3,000 pound armor stone, two layers of 250 pound underlayer stone overlaying a geotextile that separates the stone revetment from the dike core.

A breakwater has been designed for two locations on the eastern side of Parsons Island (Figures 3-3 and 3-4). One breakwater is a structure connected to Dike Section No. 1; a 2000-ft long series of segmented offshore breakwaters are located about 300 ft from the eastern shore of Parsons Island. Crest elevation of the breakwaters would be +4 ft MLLW, with two layers of 600 pound armor stone overlaying a 60 pound core layer.

5. REFERENCES

Engineer Consultation Construction Remediation, Inc. August 24, 2001. *Final Geotechnical Report for Preliminary Feasibility Study, Parsons Island, Chesapeake Bay, Maryland.*

Federal Emergency Management Agency (FEMA). 1988. Investigation and Improvement of Capabilities of the FEMA Wave Runup Model (Technical Documentation for Version2.0). Washington, D.C.

Gahagan & Bryant Associates, Inc. – Moffatt & Nichol, Engineers Joint Venture (GBA – M&N JV). 1995. *Poplar Island Restoration Project Hydrodynamic and Coastal Engineering Final Report.*

Goda, Y. 1985. *Random Seas and Design of Maritime Structures.* University of Tokyo Press.

National Climatic Data Center (NCDC). 1994. U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA). Historical Fastest Mile Winds for Baltimore, MD. Asheville, NC.

National Ocean Service (NOS). 1982. U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA). Historic Extreme Winds for the United State – Atlantic and Gulf of Mexico Coastline. Rockville, MD.

National Ocean Service (NOS). 1996. U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA). Tidal Currents 1996. Rockville, MD
Engineer Consultation Construction Remediation, Inc. (E2CR)

National Ocean Service (NOS). 1997. U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA). Tide Datums for Selected Stations.

Pilarczk, K.W. Editor. 1990. *Coastal Protection.* A.A. Balkema Rotterdam.

United Kingdom (UK) Construction Industry Research and Information Association and the Netherlands Centre for Civil Engineering Research and Codes (CIRIA/CUR). 1991. Manual on the Use of Rock in Coastal and Shoreline Engineering. CIRIA Special Publication 83, London CUR Report 154, The Netherlands.

U.S. Army Corps of Engineers (USACE). 1990. ER-1110-2-1407, Hydraulic Design of Coastal Shore Protection Projects.

U.S. Army Corps of Engineers (USACE), Waterways Experiment Station (WES), Coastal Engineering Research Center (CERC). 1984. Shore Protection Manual, Volumes I and II. Fourth Edition. U.S. Government Printing Office.

U.S. Army Corps of Engineers (USACE). 1985. EM 1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads.

Van der Meer, J.W. 1988. Rock Slopes and Cravel Beaches Under Wave Attack. Doctoral Thesis, Delft University of Technology.

Van der Meer, J. W. 1992. Wave Runup and Overtopping on Coastal Structures. *Journal of Coastal Engineering.*

Virginia Institute of Marine Science (VIMS). 1978. Storm Surge Height-Frequency Analysis and Model Prediction for Chesapeake Bay. Special Report No. 189 in Applied Marine Science and Ocean.